LESSONS FOR STEEL STRUCTURES FROM THE 2009 EARTHQUAKE DAMAGE IN PADANG

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SUMMARY

The Padang earthquake is a timely reminder to New Zealand structural engineers of a number of things with respect to seismic design and construction practice of steel structures. These include: The importance of implementing the latest seismic loadings and design technology into new and existing structures without undue delay; The need to maintain an effective Building Code enforcement and audit process, including the keeping of publicly transparent compliance records; The important role of the design engineer in observing and auditing the interpretation and implementation of the design is essential, to prevent improper substitution of materials and ill-considered design changes; The need for ongoing continuing professional development and education for design, construction and building code enforcement officials to develop and maintain technical competency; The separation of non-structural elements from interfering with the primary seismic resisting system needs to be carried through diligently from design and into construction. Where structural separation is not achieved then design models for integrating unreinforced brickwork panels within moment resisting frames need to be developed, particularly for retrofit situations; The design for weak-axis bending of two way moment resisting steel frames requires careful attention to secondary effects, and should be avoided where possible; Non-self centring structural elements need to be identified at design stage and designed to minimise inelastic behaviour during ultimate limit state earthquakes; Diagonal bracing rods should be designed to avoid failure within couplings. Consideration should also be given to the dynamic response of the roof level bracing system to heavy wall induced lateral loads; Connections at the interface of steel work with concrete and masonry sub-trades need to be carefully monitored to ensure intended design performance is achieved; Unreinforced masonry without lateral tiebacks should be avoided on lintels over egress-ways; A guide of typical structural repair methods would also be a useful tool for post-earthquake use, to quickly identify appropriate repair strategies and allow repair estimates to be developed. At a philosophical level, should a post-earthquake repair be required to simply allow a resumption of functionality? Alternatively should the repair be required to reinstate the structural performance to its pre-earthquake strength? Or should the repair improve the seismic resisting performance of the structure in line with current earthquake engineering knowledge?

INTRODUCTION

This paper initially describes general structural response to the earthquake, the importance of an effective building control system to lift the earthquake resisting capability of building stock, and the urgency of seismic strengthening of existing buildings. The performance of steel and concrete moment resisting frames with brickwork infill panels and the challenge they present for design and retrofit are discussed. Other things reviewed are: the design of non-self-centring elements such as upper storey columns supporting pitched rafter roofs; the performance of diagonal steel roof bracing; the connection of concrete and masonry to steelwork; falling hazards over egress ways; the need for a structural repair methods guide after an earthquake and the issue of how far a repair should go in terms of allowing a resumption of functionality, or reinstatement to its pre-earthquake strength or improvement of performance.

Both authors were directly involved in the early response to the Mw 7.6 earthquake in Western Sumatra on 30th September, 2009.

Clark Hyland was part of a team of ten NZSEE/EENZ volunteer structural engineers sent to Padang, Indonesia, led by Dave Brunsdon. The team was supported by and represented NZAID, and worked with the UNDP to assist the Department of Works agencies in the Padang area of Indonesia, with structural safety and reparability assessments of earthquake damaged structures.

Sugeng Wijanto assisted the NZAID team doing assessments in Padang. He was also involved with the private assessment of structures for client companies in Padang, and worked with the EERI reconnaissance team from the USA. He gained his PhD at the University of Canterbury, and runs a significant consulting engineering practice in Jakarta.

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STRUCTURAL RESPONSE TO THE EARTHQUAKE

The Mw 7.6 earthquake of 30th September, 2009 on the west coast of Sumatra affected a population of 1.2 million people. 1,195 died and there was significant damage to around 140,000 houses and 4,000 other buildings [1](EERI 2009). In Padang 383 people died and 431 were seriously injured as a result of building damage and collapse.

Records from the one strong ground motion record in the region, located 12 km away from Padang, at the base of the mountains, showed 20 seconds of strong shaking, with a peak ground acceleration of 0.3g. The structural response spectral accelerations in the short natural period range of 0.1 to 0.5 seconds varied with 1.2g to 0.8g at each end (Figure 1). When account is also taken for deep soil effects in Padang then these values will be increased further.

The high spectral accelerations for structures with natural periods of less than 0.5 seconds is one obvious explanation for the large number of two to five storey reinforced concrete and masonry infill wall structures that suffered serious damage and collapse. The 2002 Indonesian structural loadings standard [2] (Standar Nasional Indonesia, 2002), applicable prior to the earthquake has a response spectrum for design with a 0.7g acceleration for structures on firm soils with natural periods below 0.5 seconds. Prior to 2002 the Indonesian loadings standard spectral acceleration requirements set in 1987, for short period structures were two and half times less than those of the subsequent 2002 standard. Between 1970 and 1987 the design accelerations for buildings in Padang was 0.1g, used in conjunction with working stress design.

The vast majority of buildings extensively damaged in the earthquake would have been designed and constructed prior to 1987. It would also be expected that buildings designed to 2002 loadings levels would have survived the earthquake well, and many of these did perform very well. However some recent structures didn't perform well and these will be discussed in the following sections.

Current Indonesian Steel Structure Codes:

Before 1984, design guidelines for steel structures in Indonesia used *Nen 3851 TGB 1972 Steel: Technical Principles for the Design and Calculation of Building Structures*. In 1984, the new *Design Regulations for Steel Building in Indonesia* was published. It was changed in name in 1987 and 1989 and became the *Design Guidelines for Steel Building*, [3] (Public Works Department, 1987), though the contents were similar to the previous code. These design codes were adapted from the Dutch Steel Code and based on working stress design. LRFD design and ductility were introduced for designing steel buildings in 2002, with the adoption of provisions from the American Institute of Steel Construction [4]. Structural reinforced concrete design provisions were also updated in 2002 [5].

However, although the LRFD method has been published in the latest code, many structural design engineers still prefer to use working stress design.

IMPORTANCE OF A BUILDING CONTROL SYSTEM

One of the difficulties in assessing the cause of damage from the earthquake in modern structures in Padang is the lack of a publicly transparent building control system. It is therefore not clear what level of design and construction standard compliance any particular structure achieved. As a consequence it can't be assumed that a structure's performance is necessarily representative of the adequacy or otherwise of the prevailing design and construction standards at the time of its development.

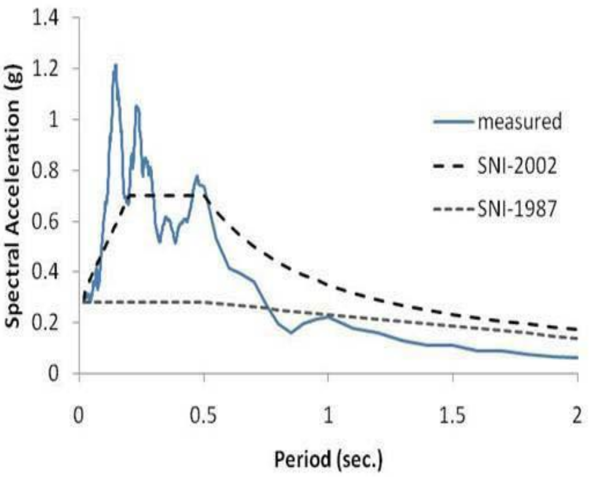

Figure 1: Actual measured response spectra (N-S component) and design response spectra [1].

The importance of maintaining an effective, knowledgeable and publicly accountable building control system to ensure application of the latest design and construction knowledge and standards is underlined by this earthquake. The Building Control system in New Zealand should therefore be appreciated and continually improved.

THE URGENCY OF SEISMIC UPGRADING

The importance of setting a level of urgency on requiring the upgrading of older building stock to current earthquake design, loadings and materials standards, is highlighted by this tragedy. The huge variation in lateral design loadings over the last forty years in Indonesia hasn't translated into seismic upgrading of existing buildings. This meant that the bulk of the building stock wasn't prepared to respond safely to the earthquake. Major loss of life and economic disruption has resulted.

In New Zealand a new urgency is needed to implement modern loadings and materials design approaches into existing structures.

THE NEED FOR CONSTRUCTION OBSERVATION BY THE DESIGN ENGINEER

The scope of the structural design engineer's observation of construction of their buildings can vary hugely in Indonesia. While the recommended practice is for weekly observation by the design engineer, this often doesn't occur particularly for construction in remote locations. Observation may be limited to once a month or less for buildings constructed in locations at some distance from the design office. The consequence of this is that wrong interpretations of the design drawings can be made by builders that are difficult to correct. Substitutions and changes are also often made without reference back to the original design engineer. The results can be disastrous when events such as an earthquake like this occur.

While New Zealand structural engineers have great respect for the skills and advice of builders, the lesson from Indonesia is that it is critical that the structural design engineer continues to be engaged to regularly observe and monitor the interpretation and implementation of the structural design by the builder. The builder and the client should not be left to make structural design changes or substitution of materials without careful consideration of the consequences by the design engineer.

THE NEED FOR CONTINUING PROFESSIONAL DEVELOPMENT

The level of understanding of engineers in Padang (and it is also believed in other regions) depends mainly on what they have learned while they studied at the university. There is little effort to keep practitioners up to date with new codes, standards and design methods. The exception is for those who are practicing in Jakarta and some of the big cities in Java.

The importance of having affordable and well put together practitioner training in New Zealand, from industry sector groups like Steel Construction New Zealand, NZSEE, SESOC and others is underlined.

MRF WITH INFILL MASONRY PANELS

Integration of MRF with Unreinforced Brickwork

A commonly observed problem in Padang was the integration of moment resisting frames (MRF) with unreinforced brick infill walls. The majority of these moment resisting frames were reinforced concrete because the use of steel construction is still developing in Indonesia and reinforced concrete is often more cost-effective. However in a few cases structural steel was used for moment resisting frames. Typically the steel columns were concrete encased in such MRF. The two major collapses of steel MRF involved this form of construction. Also an issue with both cases was the incorporation of the steel MRF structure into older concrete MRF infill structures.

Performance of MRF with Brick Infill

MRF with infilled brick panels in fact performed adequately in cases where there was a regular horizontal arrangement of infilled MRF and vertical consistency of infilling panels up the frames. Where collapses occurred with this form of construction it was typically at ground floor level where infill panels were discontinued creating a soft storey. This appears to have been one of the causes for the collapse of a new 6 level steel MRF with infill brick addition to an existing hotel. It is reported that 200 people died in its collapse.

In buildings with good horizontal and vertical regularity of in-filled MRF, the infill brick work walls were typically damaged with major diagonal cracking across the panels, but with minor or no damage to the moment resisting frames that confined the brickwork. Few examples of such brick infill panels collapsing out of plane were observed indicating the general adequacy of the confinement provided by the frame boundary elements.

While the appearance of these badly cracked brick infill panels was disconcerting, it was clear in many instances that the infilled MRF had behaved more as an unreinforced masonry shear wall with reinforced concrete or steel boundary elements, rather than as a MRF. The brick infill walls sustained damage in initial diagonal cracking, but maintained strength using compression field behaviour. The repair of unreinforced brick infill walls is a relatively low cost exercise in comparison with repairing damaged reinforced concrete or steel elements.

Current Design and Construction of MRF with Infill Brickwork

The current design code approach in Indonesia does not specifically control the effect of masonry walls that infill MRF. Generally, both in design and during construction it is considered that the brick infill panels are non-structural and do not contribute to seismic performance except by increasing the seismic mass. This leads to the frame being analysed for loadings using the natural period of a flexible frame structure rather than the much shorter natural period of a rigid shear wall structure. The frame is also designed for frame action, rather than more correctly as a shear wall, with secondary flexural and shear capacity provided by the frame. No attempt is typically made during construction to isolate the masonry walls from interfering with the lateral movement of the structural frames.

In 1982, the Indonesian Public Works Department published guidance on the design of MRF with infill brickwork. There were four types of buildings referred to as A, B, C and D types for which the infilled brick walls are taken into account as integrated parts of the building or are accounted for separately. It was disseminated through some workshops in several major cities by Andrew Charleson from Victoria University at that time. Unfortunately there has been little use of the guidelines and they have never been updated.

Figure 2: Collapsed 5 level two-way steel MRF addition to hotel.

Compression Field Design of Infill Panels

A rigorous compression field design model appears to be needed for brick infill MRF. Where brick work is not structurally separated, the structure should be analysed with the contribution of those unreinforced walls carefully considered to shorten the natural period of the structure and increase the seismic response of the structure.

While a rigorous design model has been developed for tension field steel shear walls, no suitable design model currently exists for compression field shear walls with confining MRF. The natural inclination of New Zealand engineers is to avoid the use of unreinforced infill brickwork in conjunction with MRF, but this is impractical in Indonesia at the moment due to economic and construction culture considerations.

In New Zealand much effort goes into separating non-structural wall elements from the earthquake resisting MRF structure. This means that the structure can be reliably designed for lower response accelerations associated with the longer natural period of the MRF. In Indonesia this is not clearly signalled in the design codes.

Weak Axis Bending of Steel MRF with Brick Infill

The collapse of a recently built five storey single bay width addition to an existing hotel, involved a two way steel moment resisting frame (Figure 2). The five storey addition totally collapsed under bending failure about the weak axis of what appear to have been concrete encased I-section columns. Brick infill panels are likely to have been incorporated with window openings into the outer wall line. The two way moment connections at the columns appeared to have maintained integrity, so it appears that the frame had not been able to cope with large displacements in the weak axis MRF.

In New Zealand two-way MRF are avoided for seismic purposes. The preference being to use designated one-way MRF in each direction to provide the necessary stiffness and strength. Such an approach also avoids the issue of concurrent actions at columns in two-way MRF. Secondary p-δ moment effects due to the displacement of the column heads relative to their bases must also be carefully allowed for in two-way MRF.

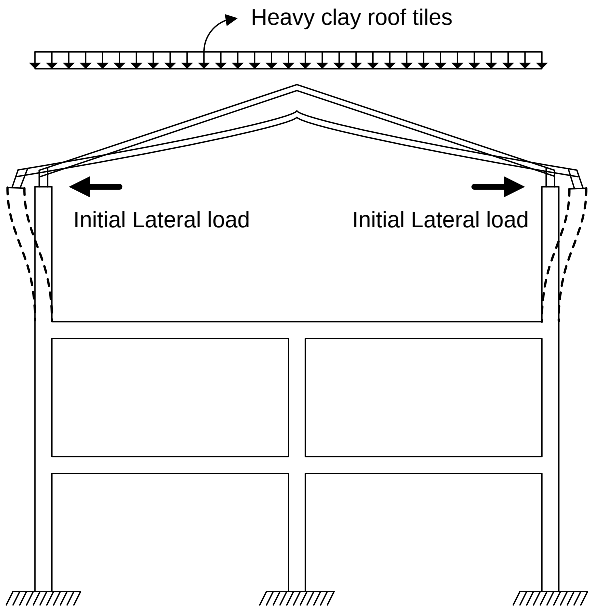

Figure 3: Reinforced concrete columns spread through shake-down of heavy tiled pitched steel roof.

DESIGN OF NON-SELF CENTRING ELEMENTS

Non-self Centring Shake-down Behaviour

Where an out of balance long term design action acts on a structural element which will act in the same direction as an earthquake action, that element will not self-centre if it deforms plastically during an earthquake. The effect of the long term action will be to accentuate the displacement in one direction and prevent it from self-centring as the structure seeks to return to equilibrium. Non-self centring shake-down can result in significant repair costs or even collapse if the displacements are high.

Upper Storey Columns Supporting Pitched Portal Rafters

A particular case of this behaviour was found in Pariaman where the upper level reinforced concrete columns supporting a simply supported pitched portal roof with heavy tile roofing, had spread outwards at opposing wall faces (Figure 3). If the pitched rafter had been tied then there would have not been any lateral dead load thrust at the column heads to cause the non-self centring behaviour. This would have been architecturally unappealing. The alternative would have been to design the cantilevered columns to act elastically in response to the seismic "parts and portions" actions loads combined with the dead load lateral thrusts imposed by the rafter.

Figure 4: Collapsed roof level rod bracing in warehouse.

DIAGONAL ROOF BRACING

Failure Hierarchy in Bracing Rods

Diagonal rod roof bracing collapsed in a portal framed warehouse with high masonry walls. The rod bracing had broken free from the rafters after failure of the weld connecting the rod bracing plate to the rafter (Figure 4). The weld appeared to be a site weld, raising the issue of how to ensure site welding quality when performed at height. From a design perspective the weld should never be the weakest link in a primary structural member such as a roof bracing rod. The preferred hierarchy of failure is that ductile elongation of the rod itself protects against connection failure at the rod ends. The rod connection therefore needs to be able to cope with the over-strength demand of the rod.

Avoid Hooked Rod Tensioners

Hooked end rod tensioners were found to have jumped free of roof bracing in another building. This reinforces the advice not use such tensioners, as roof bracing rods can undergo cycles of tension and slackening as an earthquake progresses, allowing the possibility for the tensioner to jump free.

Effectiveness of Pitched Roof Bracing with Heavy Walls

A question arising from the collapse of the roof bracing in the warehouse discussed above, is how to effectively design in-plane roof bracing in pitched roofs to restrain the out of plane reactions from laterally supported masonry or precast panels. Where the portal legs are slender or the rafters are supported directly from the wall panel heads particular care is required to understand the dynamic behaviour of the roof and walls, and to resolve the actions into the bracing.

CONNECTIONS OF CONCRETE AND MASONRY TO STEELWORK

***Figure 5:** Wall panel detached from roof steelwork above drive way.*

Care at Sub-Trade Interfaces

The weakest link in construction can often be found at the inter-face between where one sub-trade ends and the next starts. The responsibility for quality can be blurred where something is not prepared adequately for the following trade to connect into within their preferred tolerances. This is a common feature in New Zealand construction around holding down bolt and other steelwork to concrete connections. Tolerance specifications for the two trades are sometimes contradictory. While this often leads to sub-contract disputes, not seen by the design engineer or client, the true victim is often quality and sub-standard earthquake performance at the interface. This issue is not unique to New Zealand construction as illustrated by the following example.

Wall Panel to Roof Steelwork Connections

Concrete masonry panels around the top storey of a three storey office building detached from the roof steelwork in a number of locations around the perimeter of the building (Figure). The panels were found to be in danger of falling onto the access-way below. It wasn't clear how the panels had been attached to the roof steelwork which was still in good condition.

FALLING HAZARDS OVER EGRESSWAYS

The example of the detachment of wall panels from roof steelwork discussed previously also highlights the need for special care to prevent falling hazards over safety egress ways in and around buildings.

Brickwork supported on steel lintel beams broke free under face loading over exit ways in many cases and could have been a cause of serious injury of those escaping the buildings (Figure). These instances reinforced the requirement in New Zealand to tie back free standing brick veneers against face loading damage.

STRUCTURAL REPAIR METHODS GUIDE

***Figure 6:** Brick infill panel on steel lintel typical of the type that collapsed under earthquake face loading.*

The development of some recommended repair concepts for reinforced concrete and steel structures in a guide able to be used immediately post-earthquake would help structural engineers and quantity surveyors to quickly identify appropriate repair strategies and cost budgets, to quickly achieve a return to economic and social functionality of damaged structures.

POST-EARTHQUAKE REPAIR OR STRENGTHENING?

The NZSEE team were faced with the need to identify potential repair concepts in the process of assessing the feasibility of repairing damaged structures. The difference between applying a repair to reinstate a damaged building to its pre-earthquake state and one to strengthen and improve the seismic performance become apparent.

What criteria should be used to decide if an earthquake damaged building be required to allow resumption of functionality, or to be strengthened to improve performance or just be reinstated to its previous state? The repair needed for each objective will typically be different. However in the absence of a legal imperative, the lowest level of repair to achieve a resumption of functionality is most likely to be all that is ever undertaken.

CONCLUSION

The Padang earthquake is a timely reminder to New Zealand structural engineers of a number of things with respect to seismic design and construction practice of steel structures.

These include:

1. The importance of implementing the latest seismic loadings and design technology into new and existing structures without undue delay.

2. The need to maintain an effective Building Code enforcement and audit process, including the keeping of publicly transparent compliance records.

3. The role of the design engineer in observing and auditing the interpretation and implementation of the design is essential, to prevent improper substitution of materials and ill-considered design changes during construction.

4. The need for ongoing continuing professional development and education for design, construction and building code enforcement officials to develop and maintain technical competency.

5. The separation of non-structural elements from interfering with the primary seismic resisting system, needs to be carried through diligently from design and into construction. Where structural separation is not achieved then design models for integrating unreinforced brickwork panels within moment resisting frames need to be developed, particularly for retrofit situations.

6. The design for weak-axis bending of two way moment resisting steel frames requires careful attention to secondary effects and should be avoided where possible.

7. Non-self centring structural elements need to be identified at design stage and designed to minimise inelastic behaviour during ultimate limit state earthquakes.

8. Diagonal bracing rods should be designed to avoid failure within couplings. Consideration should also be given to the dynamic response of the roof level bracing system to heavy wall induced lateral loads.

9. Connections at the interface of steel work with concrete and masonry sub-trades needs to be carefully monitored to ensure intended design performance is achieved.

10. Unreinforced masonry without lateral tiebacks should be avoided on lintels over egress ways.

11. A guide for typical structural repair methods would be a useful tool for post-earthquake use, to quickly identify appropriate repair strategies and allow initial repair cost budgets to be developed.

12. It needs to be decided before a major earthquake what criteria are used to set the level of repair required after an earthquake.

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